SEISMIC VULNERABILITY AND STRENGTHENING OF A MODERN ARCHITECTURE BUILDING

H. Varum, A. Costa, H. Rodrigues
Civil Engineering Department, University of Aveiro, Portugal,
hvarum@civil.ua.pt; acosta@civil.ua.pt, hrodrigues@aveiro-norte.ua.pt

RESUMEN

En Portugal, después de la segunda gran guerra mundial, emergió una nueva generación de arquitectos, influenciados por el movimiento de Arquitectura Moderna. En los años cincuenta fue construido un gran número de edificios inspirados en los principios de la Arquitectura Moderna en Lisboa, con características estructurales particulares, que en ciertas condiciones pueden introducir un comportamiento estructural deficiente, especialmente para acciones sísmicas. Como ejemplo, los edificios apoyados en "pilotis", muy frecuentes en este estilo arquitectónico, presentan mecanismos de comportamiento tipo soft-storey, que hacen estas estructuras muy vulnerables en relación a las acciones sísmicas. Los modelos numéricos disponibles, en conjugación con las normativas y recomendaciones internacionales, viabilizan la evaluación de la seguridad de los edificios existentes. Para evaluar la vulnerabilidad de esta topología estructural fue estudiado un edificio representativo de la Arquitectura Moderna, en Lisboa. El edificio fue estudiado con recurso a un programa de análisis no-lineal dinámica, PORANL, que permitió evaluar la seguridad sísmica para varios niveles de acción, de acuerdo con metodologías y recomendaciones recientes. Finalmente, se propone y analiza una solución de refuerzo estructural, basada en un dispositivo de disipación de energía acoplado a un sistema de contraviento metálico, de forma a mejorar el comportamiento sísmico del edificio estudiado.

SUMMARY

In Portugal, at the end of the World War II, a new generation of architects emerged, influenced by the Modern Movement Architecture. In the fifties, it was built a large number of Modern housing buildings in Lisbon, with particular structural characteristics that, in certain conditions, can induce weaknesses in structural behaviour, especially under earthquake loading. For example, the concept of buildings lifted in "pilotis", present in this architectural style, can strongly facilitate the occurrence of soft-storey mechanisms, which makes these structures very vulnerable to earthquake actions. To investigate the vulnerability of this type of construction, one building, representative of the Modern Architecture, in Lisbon, was studied with a non-linear dynamic analysis program, PORANL, which allows the safety evaluation according to recently proposed assessment procedures. Additionally, it was proposed and analyzed a retrofitting solution, to improve the seismic performance, based on a bracing system with a damping device associated.

Introduction and description of the studied structure

The study of the seismic vulnerability of existent buildings in urban areas with moderated/high seismic risk is of extreme importance, to evaluate its safety according to the recently proposed international codes and recommendations. The high number of buildings constructed in Lisbon, Portugal, in the fifties, with particular modern architecture style and characteristics reveal a deficient seismic behaviour. In this work it was studied the seismic vulnerability of an existing building representative of the modern architecture style. The building under study is located in the western part of the Lisbon.In the block relationship with the ground, there is a clear reference to the 1927 Le Corbusier point o architecture – "the house assents in pilotis"; witch is assumed with big formal value. The block plan is rectangular with 11.10m width and 47.40m length (figure 1). The building has the height of 8 habitation storeys plus the pilotis height at the ground floor. The "free plan" is also a reference because the house was conceived in a way of flexibility in use. But, the 12 structural plane frames define the architectural plan of the floor type, with 6 duplex apartments. The distance between frame's axes is 3.80m. Each frame is supported by two columns and has one cantilever beam on each side with 2.80m span.~

Figure 1. General views of the building block under analyses

The twelve transversal plane frames have the same geometric characteristics for all beams and columns. However, three different frame-types were identified, according to reinforcement detailing. A peculiar structural characteristic of these type of buildings, with direct influence in the global structural behaviour, is the empty ground storey, without infill masonry walls. Furthermore, at the ground storey the columns are 5.5m height. All the upper storeys have an inter-storey height of 3.0m. Therefore, it is introduced a soft-storey mechanism at these level.

In the numerical models for the analysis of the building in the two independent directions (X and Y), it was considered a concrete slab with 1.25m width and 0.20m thick. A detailed definition of the existing infill panels, in terms of dimensions, location and materials were considered in the structural models (Figure 2).

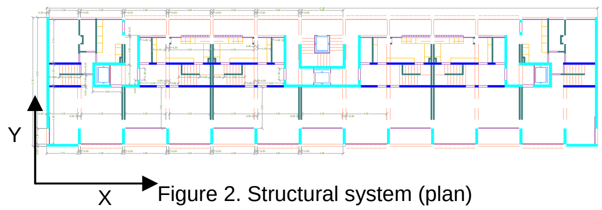


Figure 2. Structural system (plan)

For the building analysis in the transversal direction (Y), it was assumed an equivalent model defined as the association of the three frame-types, interconnected by rigid strut bars, as showed in figure 3. In this global model, the geometric and mechanical characteristics of each frame are multiplied by the number of occurrences of each frame-type. For the analysis in the longitudinal direction (X), and because the double symmetry in plan, it was studied just one quart of the building. For the global model results a six columns structure linked at all storey levels by the RC slabs. No full-bay infill panels exist in the longitudinal direction. Therefore, an external simplified global infill masonry model was considered, as represented in figure 4, connected, at storey levels, throw rigid struts to the RC structure.

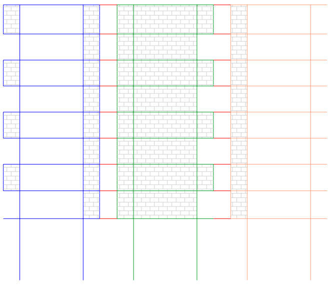


Figure 3. Structural system for transversal direction (Y)

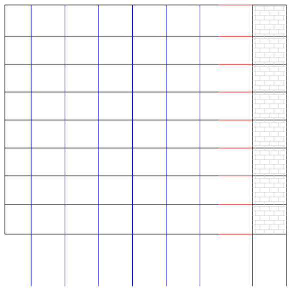


Figure 4.Structural system for longitudinal direction (X)

Models description

Nowadays, in the analysis of structures subjected to seismic actions, the use of non-linear behaviour laws and hysteretic rules reveals a great advantage, because it makes possible a more rigorous representation of the seismic structural response. To simulate the structural behaviour of the building presented in the previous sections, it was used a computer program PORANL, that contemplates the non-linear bending behaviour of RC elements (beams and columns) and the influence of the infill masonry panels in the global response of the buildings. Each RC structural element is modelled by a macro-element defined by the association of three bar finite elements, two with non-linear behaviour at its extremities (plastic hinges), and a central element with linear behaviour (figure 5).

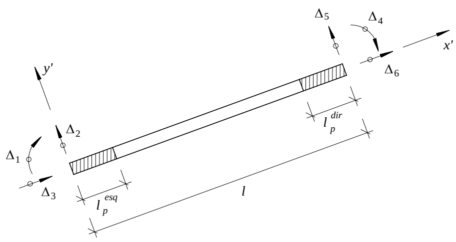


Figure 5. Frame macro-element (Varum, 1996)

The non-linear monotonic behaviour curve of a cross-section is characterized through a tri-linear moment-curvature relationship, corresponding respectively to: the initial non-cracked concrete, concrete cracking and steel reinforcement yielding (Varum, 1996). The monotonic curve is obtained using a fibre model procedure (see figure 6), from: the geometric characteristics of the cross-sections, reinforcement and its location, and material properties. The non-linear behaviour of the plastic hinge elements is controlled through a modified hysteretic procedure, based on the Takeda model, as illustrated in figure 7. This model developed by Costa (1989) represents the response evolution of the global RC section to seismic actions and contemplates mechanical behaviour effects as stiffness and strength degradation, pinching, slipping, internal cycles, etc.

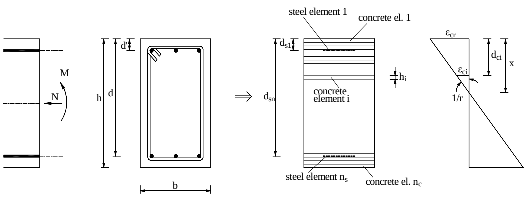


Figure 6. Fibre model for RC elements

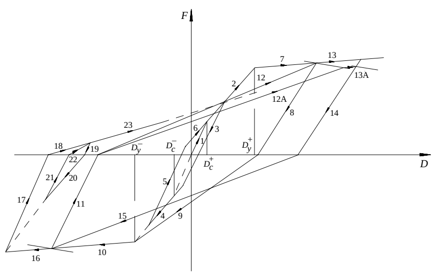


Figure 7. Hysteretic model for RC elements

To represent each infill masonry panel, an improved macro-model, based on the bi-diagonal equivalent strut model, is used (figure 8). The proposed macro-model was implemented in the non-linear structural analysis program PORANL (Rodrigues et al., 2005). The macro-model adopted represents the non-linear behaviour of an infill masonry panel and its influence in the global RC structural behaviour under static or dynamic loading.

The monotonic behaviour curve of each panel depends on the panel dimensions, eventual openings (dimensions and position), material properties (bricks, mortar, and plaster), and quality of the handwork, interface conditions between panel and the surrounding RC elements. The non-linear behaviour of the infill masonry panels subjected to cyclic loads is controlled through an hysteretic procedure and rules, illustrated in figure 9, and represents mechanical effects as stiffness and strength degradation, pinching, and internal cycles.

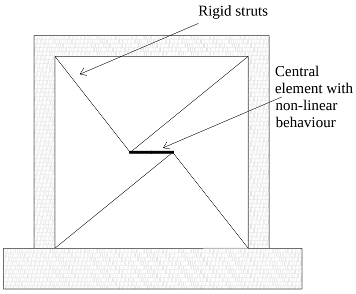


Figure 8. Infill masonry panel macro-model

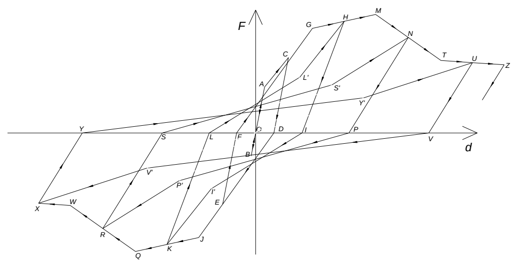


Figure 9. Hysteretic model for infill masonry panels

Static loads, masses and damping

For the numerical analyses, constant vertical loads distributed on beams were considered in order to simulate the dead load associated to the self-weight of RC structural elements, infill walls, finishing's, and the correspondent quasi-permanent value of the live loads, totalising a value of 8.0kN/m2. The mass of the structure was assumed concentrated at storey levels. Each storey has a mass, including the self-weight of the structural and non-structural elements, infill walls and finishings, and the quasi-permanent value of the live loads, of about 4Mtons. For the dynamic analysis, the storey mass is assumed to be uniformly distributed on the floors. For each structural model, a Rayleight damping matrix, with 1% damping ratio for the first two natural modes, was considered.

Natural frequencies and modal shapes

A first validation of any structural numerical model can be achieved comparing the experimentally measured and the analytically estimated natural frequencies. In table 1 are listed the four first natural frequencies computed, for the building and for each direction (X and Y). To validate the numerical building models, in the two independent directions, it were measured the first natural structural frequency, with a seismograph and for the ambient vibration. The measured first frequency for each direction is indicated, in brackets, in table 1. A good agreement was found between the experimentally measured frequencies (1.17Hz for the longitudinal direction and 1.56Hz for the transversal direction) and the frequencies estimated with the structural numerical models (1.08Hz for the longitudinal direction and 1.75Hz for the transversal direction), which constitutes the first validation of the numerical model developed. In figure 10 are represented the first natural mode shapes calculated for each direction. From the analysis of the shapes of the first vibration modes, in both directions, it is clear that seismic actions will induce a soft-storey mechanism response. This conclusion will be confirmed with the earthquake result analysis in the forwarding sections.

Table 1: Natural frequencies for directions X and Y

Frequencies	Direction	
	Longitudinal X (Hz)	Transversal Y (Hz)
1st	1.08 (1.17)	1.75 (1.56)
2nd	5.67	6.41
3rd	6.32	8.14
4th	8.10	8.80

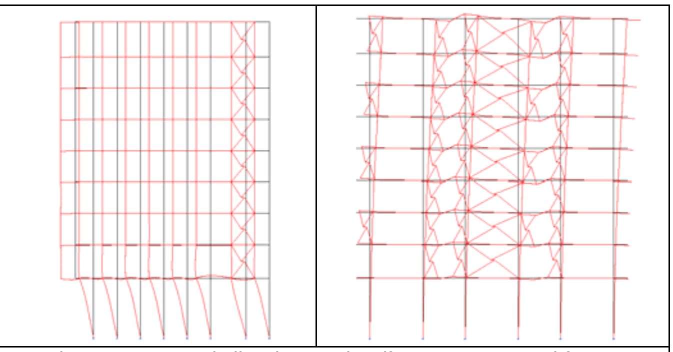


Figure 10. Natural vibration modes (f1,X = 1.08Hz and f1,Y = 1.75Hz)

Earthquake Input Signals

Three artificial earthquake input series were adopted for the seismic vulnerability analysis of the building. The first series (A) was artificially generated for a medium/high seismic risk scenario in Europe (Carvalho et al., 1999), for various return periods (table 2). The second and third series (B and C, respectively) were generated with a finite fault model for the simulation of a probable earthquake in Lisbon (Carvalho et al., 2004), calibrated with real seismic actions measured in the region of Lisbon. The earthquakes of the B and C series were scaled to the peak ground acceleration of series A, for each return period. In table 2 are presented the peak ground acceleration and the corresponding return period for each earthquake's intensity.

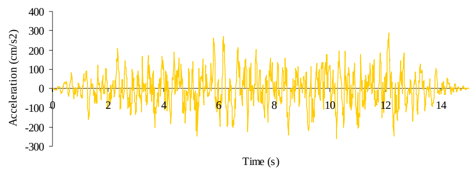


Figure 11. Accelerogram A

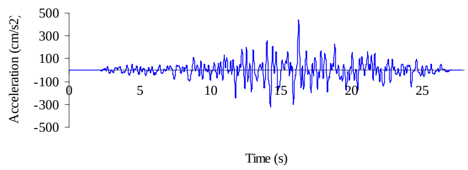


Figure 12. Accelerogram B

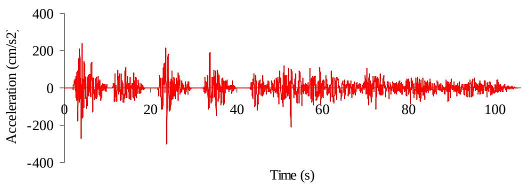


Figure 13. Accelerogram C

Table 2: Reference earthquake action (peak ground acceleration and corresponding return period)

Return period (years)	Peak ground acceleration (×g)
73	0.091
475	0.222
975	0.294
2000	0.380
3000	0.435
5000	0.514

Results Analysis

As observed in the free vibration shape modes, the structural response of the building, in both directions, clearly induces soft-storey mechanism behaviour (at the ground floor level). This structural behaviour leads to large storey deformation demands at the first storey, while the upper storeys remain with very low deformation levels. In figure 14 are illustrated, for the longitudinal direction, the numerical results in terms of envelop deformed shape, maximum inter-storey drift, and maximum storey shear, for each earthquake input motion of the series A. The envelope for the transversal direction is similar to the longitudinal direction.

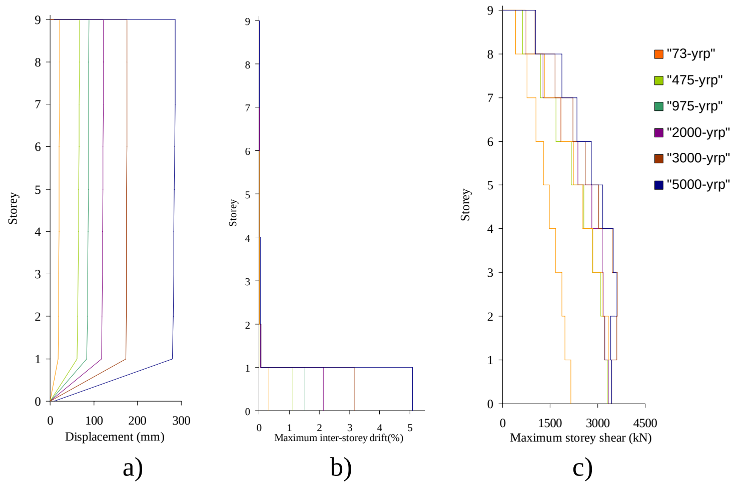


Figure 14. Results for the longitudinal direction (X) and earthquakes of the series A: a) envelop deformed shape; b) maximum inter-storey drift profile; c) maximum storey shear profile

From the analysis of the results in terms of building envelop deformed shape and inter-storey drift profile, for both directions, it can be concluded that the deformation demands are concentrated at the first storey level. In fact, the absence of infill masonry walls at the ground storey and the larger storey height (5.50m for the 1st storey and 3.00m for the upper storeys), induces an important structural irregularity in elevation, in terms of stiffness and strength.

For all the structural elements (columns and beams), and for all the seismic input action levels, the shear force demand assumes a value inferior to the corresponding shear capacity, which confirms its safety in shear.

Vulnerability curves

In this section are compared, for the three earthquake series of input motions, the vulnerability curves in terms of maximum drift at ground storey, maximum 1st storey shear and maximum top displacement, for the longitudinal and transversal directions. In figures 15-a and 16-a are plotted the vulnerability curves, for the longitudinal and transversal directions, in terms if the maximum 1st storey drift, obtained from the numerical analysis. Results show that, for the 1st storey, the maximum inter-storey drift demand for the longitudinal direction is larger than for the transversal, being the most vulnerable the longitudinal direction of the building. In figures 15-b and 16-b are represented the vulnerability curves in terms of maximum 1st storey shear force. In figures 15-c and 16-c are represented the obtained vulnerability curves in terms of maximum top displacement. Shear demand at 1st storey does not increase for earthquake input actions larger than the corresponding to the return period of 475 years, inducing demands increasing just in terms of deformation, as can be observed in the results in terms of 1st storey drift and top displacement.

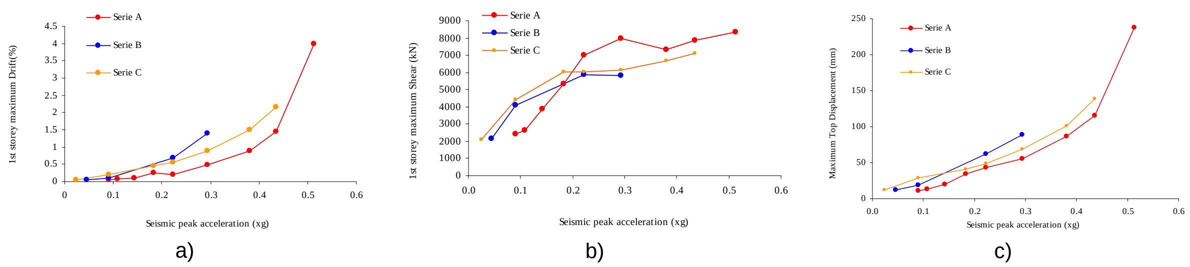
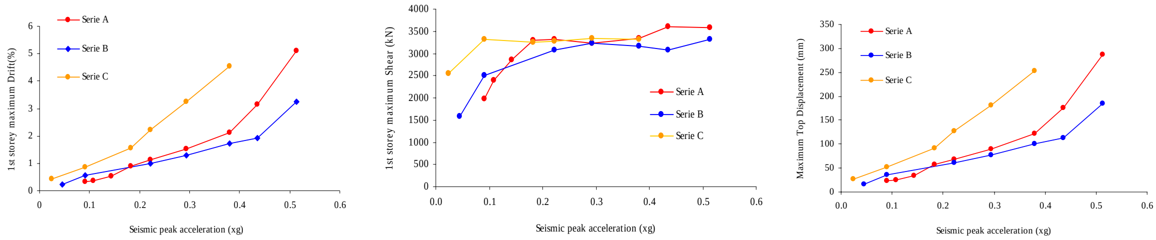


Figure 15. Results for the transversal direction (Y): a) 1st storey maximum drift; b) 1st storey maximum shear; c) Maximum top displacement



a) b) c)

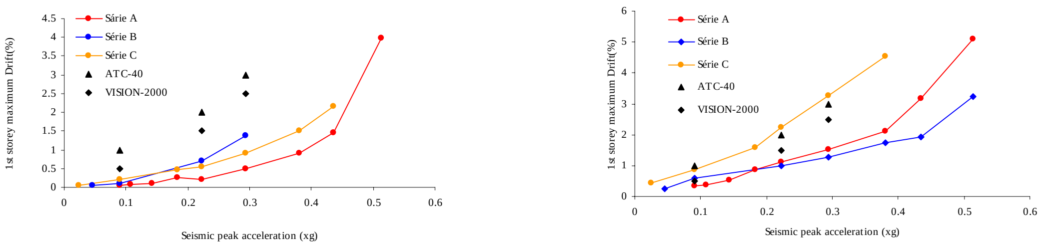
Figure 16. Results for the longitudinal direction (X): a) 1st storey maximum drift; b) 1st storey maximum shear; c) Maximum top displacement

Building seismic safety assessment

As presented in previous sections, for each direction (X and Y), the building structure was analysed for three series of earthquakes, in order to estimate deformation demands, and consequently damage levels for each input earthquake intensity. The obtained results allow verifying the safety according to the specified hazard levels, for example, the proposed in VISION-2000 (1995) and ATC-40 (1996) recommendations. In table 3 are presented the acceptable inter-storey drift limits, for each structural performance level, according to the ATC-40 and in VISION-2000 proposals, respectively. In figure 17 are represented the vulnerability functions in terms of maximum 1st storey drift, already presented in the previous section, with indication of the safety limits proposed at the ATC-40 (1996) and VISION-2000 (1995) recommendations (as summarised in table 3). Comparing the maximum storey drift demands with the safety limits proposed at the ATC-40 and VISION-2000 recommendations, it can be concluded that the building safety is guaranteed in the transversal direction (Y), for the three earthquake input series considered. For the longitudinal direction (X), the safety is guaranteed for earthquake series A and B, but not for C series.

Table 3: Inter-storey drift limits according to: ATC-40 (1996), and, VISION-2000 (1995)

	ATC-40 (1996)				VISION-2000 (1995)			
Performance Level	Immediate Occupancy	Damage Control	Life Safety	Structural Stability	Fully Operational	Operational	Life Safe	Near Collapse
Drift Limit	1%	1-2%	2%	$0.33\frac{V_i}{P_i} \approx 7\%$	0.2%	0.5%	1.5%	2.5%



a) b)

Figure 17. Maximum 1st storey drift vs. peak acceleration and safety limits: a) transversal direction – Y b) longitudinal direction – X

Proposed retrofitting solution

For the improvement of the seismic response of the building under study it was analysed a retrofitting solution, intending to reduce the soft-storey mechanism. This solution aims to reduce the deformation demand at the ground floor level. More specifically, it was proposed a x-bracing system with a shear-link dissipation device associated (see figures 18, 19 and 20), which can increase stiffness and damping of the building, consequently reduce the deformation demands. This retrofitting solution is based on a solution studied by Varum

(2003). The adoption of a x-bracing retrofitting solution was proposed due to the efficiency in reducing the deformation demands of the building, and on other hand due to this retrofitting solution do not changes significantly the architecture (only applied at the ground floor) (see figures 19 and 20). Many alternatives for the location of the bracings can be chosen, namely in the central or external bays (see figures 19 and 20). It was developed and implemented a new numerical model, in the computational program (VisualANL) to simulate the non-linear behaviour of the device. The proposed model was implemented in the computer program and calibrated with experimental results on a full-scale cyclic test in a frame retrofitted with the same dissipative device (Varum, 2003). The hysteretic behaviour and rules and the results of the calibration analysis are presented in figures 21 and 22, respectively.

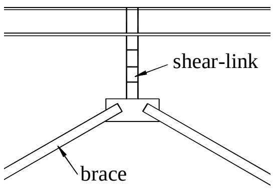


Figure 18. Shear-link (energy dissipation)

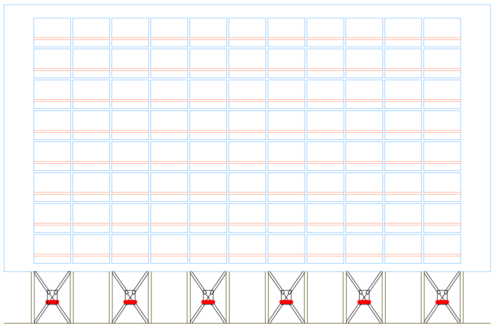


Figure 19. Location of the shear link in the longitudinal direction

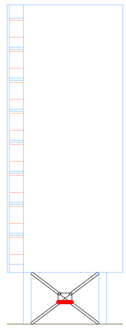


Figure 20. Location of the shear link in the transversal direction

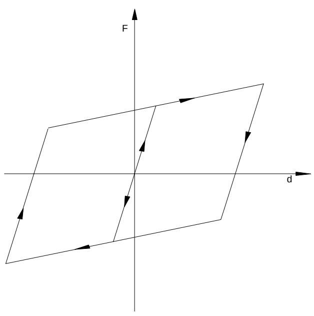


Figure 21. Hysteretic behaviour of the shear-link implemented in VisualANL

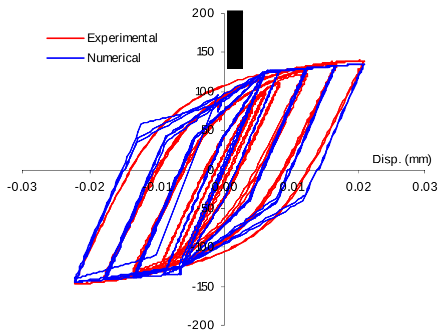
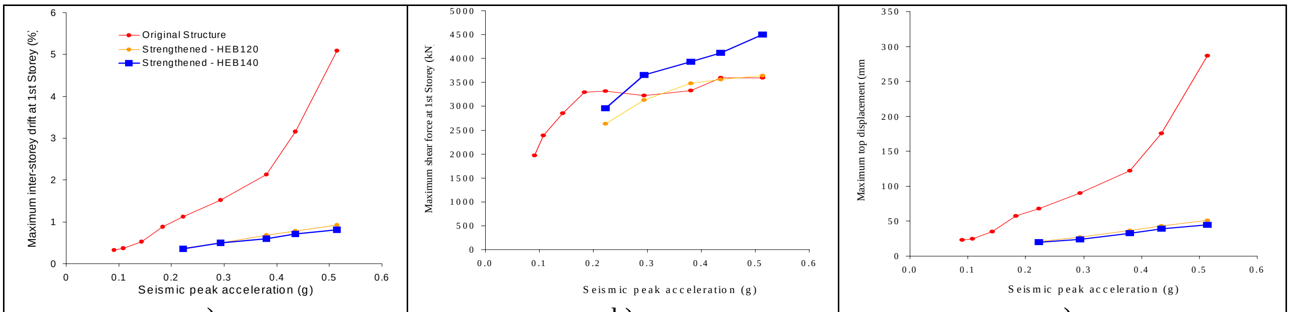


Figure 22. Calibration of the numerical model for the device cyclic behaviour

With the proposed solution for the longitudinal direction were tested two different solutions (Solution A: HEB140, L = 60 cm and Solution B: HEB120, L = 60 cm) The results were evaluated in terms of maximum 1st storey drift, maximum base shear and maximum top displacement as presented in Figure 23. However from the observation of the first results some conclusion can be taken about the Retrofitting efficiency: i) The is a pronounced reduction of the inter-storey drift and top-displacement demands and a similar deformation demands reduction level for both solutions, ii) increased global shear demands for shear-link HEB140, iii) soft-storey mechanism is prevented, even for significant acceleration levels (0.5g). However, the strengthening solution proposed must be optimized in terms of number and location of the shear-links, and internal properties as strength and elastic stiffness



a) b) c)

Figure 23. Results for the transversal direction (Y): a) 1st storey maximum drift; b) 1st storey maximum shear; c) Maximum top displacement

Concluding remarks

The global structural safety of a modern architecture building at the Infante Santo Avenue was investigated. Although the results indicate the building safety for the Basic Objectives according to the international seismic recommendations (ATC-40 and VISION-2000), it should be pointed out that additional analyses have to be performed. The input motion earthquakes adopted for these analyses can be not fully representative of the possible seismic action in Lisbon. In other way, the level of structural damage does not dependent just of the peak ground acceleration of the earthquake. Additional analyses should be performed using other earthquake motions.

Shear capacity was verified for all the input motions. However, the model adopted for these analyses does not consider the geometric non-linearity, which can increase significantly the moments in columns and global storey lateral deformations (drifts). Therefore, to guarantee the seismic safety verification of the building, it is judged focal to verify the results using a model that considers the geometrical non-linearities. Finally, it was proposed a simple and economic seismic retrofitting solution which is able to reduce the seismic vulnerability associated to particular structural behaviour and deficiencies of typical existing buildings of modern architecture style as showed by the first results, however, further analyses to optimize the efficiency of the proposed retrofitting solution should be performed.

Acknowledgements

This paper (or document) reports research developed under financial support provided by "FCT - Fundação para a Ciência e Tecnologia", Portugal

References:

- Tostões, A., Os verdes anos na arquitectura Portuguesa dos anos 50, Porto, FAUP, 1997 (in Portuguese)
- Le Corbusier, Vers une architecture, Paris, Flammarion, 1923
- Miranda, L., Rodrigues, H., Fonseca, J. Costa, A., Relatório de inspecção ao Bloco 3 do complexo do Infante Santo, 2005 (in Portuguese)
- Varum, H., Modelo numérico para análise de pórticos planos de betão armado, MSc Thesis, Civil Engineering Department, University of Porto, 1996 (in Portuguese)
- Costa, A.G., Análise sísmica de estruturas irregulares, PhD Thesis, Civil Engineering Department, University of Porto, 1989 (in Portuguese)
- Rodrigues, H.; Varum, H.; Costa, A., Numeric model to account for the influence of infill masonry in the RC Structures Behaviour, Congreso Métodos Numéricos en Ingeniería, 2005
- Carvalho, E.C.; Coelho, E.; Campos-Costa, A., Preparation of the full-scale tests on reinforced concrete frames - Characteristics of the test specimens, materials and testing conditions. ICONS report, Innovative Seismic Design Concepts for New and

Existing Structures, European TMR Network - LNEC, Lisbon, 1999

- Carvalho, A., Campos-Costa, A., Oliveira, C.S., A stochastic finite–fault modelling for the 1755 Lisbon earthquake, 13th World Conference on Earthquake Engineering Vancouver, B.C., Canada, 2004
- SEAOC, Performance based seismic engineering of buildings, Part 2: Conceptual framework - Vision 2000 Committee, Structural Engineers Association of California, Sacramento, California, 1995
- ATC-40, Seismic evaluation and retrofit of concrete buildings - Applied Technical Council, California Seismic Safety Commission, Report No. SSC 96-01 (two volumes), Redwood City, California, US, 1996
- Varum, H., Seismic assessment, strengthening and repair of existing buildings, PhD Thesis, Department of Civil Engineering, University of Aveiro, 2003